Effect of Polypropylene Fiber Reinforced Cement Composites on the Seismic Performance of Bridge Columns based on Full Scale and Scaled Model Experiments

Richelle G. Zafra[a], Kazuhiko Kawashima[b], Tomohiro Sasaki[c], Koichi Kajiwara[c] and Manabu Nakayama[c*]

[a] Department of Civil Engineering, University of the Philippines Los Baños, College, Laguna 4031, Philippines
[b] Department of Civil Engineering, Tokyo Institute of Technology, Meguro , Tokyo 152-8552, Japan
[c] Hyogo Earthquake Engineering Research Center, National Research Institute for Earth Science and Disaster Prevention, Japan

Abstract

During a strong earthquake, extensive damage of bridge columns usually occurs at the plastic hinge region. To mitigate the damage, the effect of using polypropylene fiber reinforced cement composites at the column plastic hinge is investigated based on full scale shake table experiments and scaled model experiments. The full scale column was subjected to the near-field ground motion recorded at the JR Takatori station during the 1995 Kobe, Japan earthquake while the model column was subjected to quasi-dynamic loading. Experimental results show that use of PFRC substantially mitigated cover and core concrete damage and local buckling of longitudinal bars. The damage sustained by the columns using PFRC was much less than the damage of regular reinforced concrete columns.

Peer-review under responsibility of organizing committee of The Narotama International Conference on Civil Engineering 2015 (NICCE-2015).

Keywords: Bridge columns; polypropylene fiber reinforced cement composites; full-scale experiments; scaled-model experiments.

1. Introduction

Recent earthquakes have caused collapse or severe damage of major bridges that were nominally designed for seismic forces. Such events led to increased research activity on seismic design of new bridges, assessment and retrofit of older substandard bridges, and the revision of seismic design philosophy [1]. Currently, the design for new bridges places emphasis on providing adequate displacement and ductility capacity with less emphasis on strength capacity so that critical damages would be prevented [2]. To mitigate seismic damage, ductile materials are currently being developed and their feasibility being investigated. High performance fiber reinforced cement composites (HPFRCC), materials produced by strengthening cement-based matrix with short, discontinuous and randomly distributed fibers has recently been used. In these materials, the addition of fibers to the brittle cement matrix causes fiber-bridging or transferring of stress across the crack resulting in a damage-mitigating behavior. Hence, HPFRCC resists the crack growth better relative to plain concrete. Nowadays, the design of HPFRCC aims at strain-hardening behavior - a plastic deformation prior to ultimate strength - for ductile failure to occur. A type of HPFRCC that is now being widely used for structural and retrofit applications is called engineered cementitious composites (ECC). ECC has high tensile strain capacity (1-8% in general) with moderate fiber dosage under usual strength matrix. Such efficient high performance results from closely-spaced multiple cracking prior to widening of a critical crack - behavior that is absent in conventional fiber reinforced concrete [3]. The formation of multiple fine cracks upon loading

* Corresponding author. Tel.: +62–877–8013–1509; fax: +62-031-593-1213.
E-mail address: fredy@narotama.ac.id

in tension leads to improvement in toughness, fatigue resistance, and deformation capacity [4; 5]. At present, HPFRCCs are used in the plastic hinge region of flexural members such as beams, columns, structural wall bases, and in members with shear-dominated response such as beam-column connections, squat walls and coupling beams [3; 5; 6]. Moreover, ECC with polyvinyl alcohol (PVA) fibers was used in one of the bents of a 1/4-scale, four-span bridge model subjected to shake-table excitations [7]. HPFRCCs have also been used for seismic retrofit applications such as dampers [8], infill panel walls [9] and concrete jacket [10]. The deformation capacity and energy absorption capacity of structural members were significantly improved. This study investigates the effect of PFRC for enhancing the damage control and ductility capacity of a full-scale bridge column (C1-6 column) subjected to a near-field ground motion based on shake table experiments [11] and scaled model (HPF-S column) experiments based on seismic response loading [12]. Knowing the effects of this material on column response can give engineers greater flexibility in designing and rehabilitating column structures and in improving their performance during earthquakes.

2. Overview of C1-6 and HPF-S Column Experiments

2.1. Column Configuration and Material Properties

C1-6 column is a full-scale column while HPF-S column is a 6/35 geometrically scaled model of C1-6. Their structural details are shown in Figs. 1 and 2, respectively.

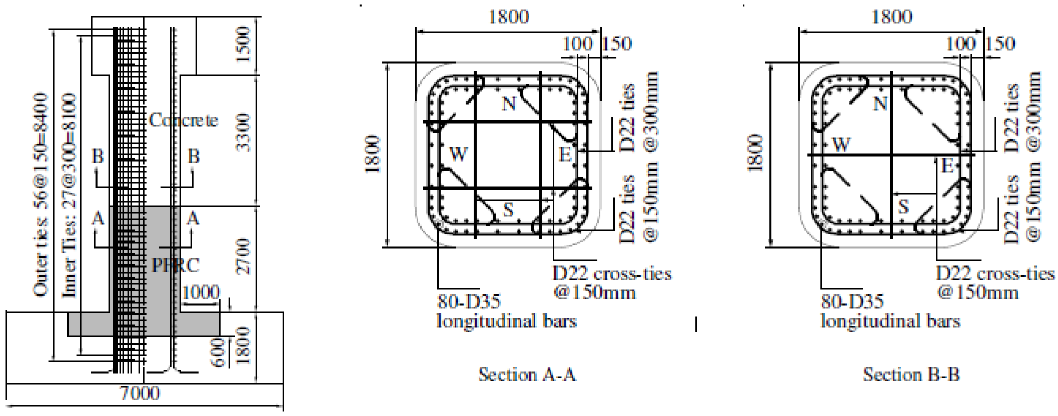


Fig. 1. C1-6 column configuration and dimensions (mm)

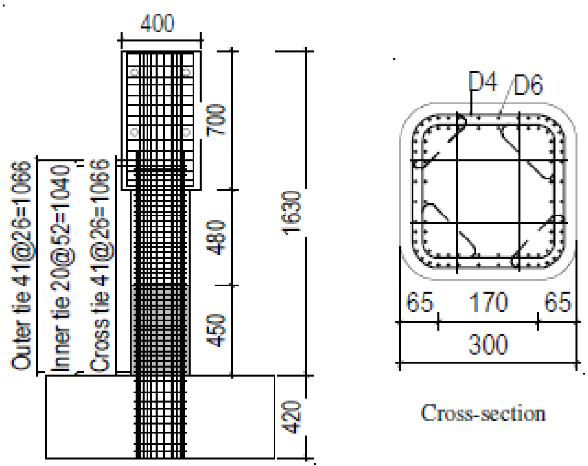


Fig. 2. HPF-S column configuration and dimensions (mm)

ISBN : 978-602-72437-2-9

C1-6 is a 7.5 m tall, 1.8 m by 1.8 m square cantilever column designed in accordance with the 2002 Japan Specifications of Highway Bridges assuming moderate soil condition under the Type II design ground motion (nearfield ground motions). PFRC was used at a part of the footing with a depth of 0.60 m below the column base and a depth of 2.7 m above the column base. The 2.7 m depth of PFRC is three times the estimated plastic hinge length of one-half the column width [13] corresponding to 0.90 m so that failure at the PFRC-concrete interface can be avoided. The 0.60 m depth of PFRC at the footing was also provided to minimize damage in the column-footing connection. The remaining portions of the column were made of ordinary concrete.

PFRC with design compressive strength of 40 MPa was used. The mixture consists of cement mortar, fine aggregates with maximum grain size of 0.30 mm, water, and 3% volume of polypropylene fibers. Monofilament polypropylene fibers with diameter of 42.6 μm, length of 12 mm, tensile strength of 482 MPa, Young’s modulus of 5 GPa and density of 0.91 kg/m3 were used. Superplasticizers were added to improve the workability of the mix. The design compressive strength of ordinary concrete used in the other parts of the column was 30 MPa.

Eighty-35 mm diameter deformed longitudinal bars were provided in two layers corresponding to a reinforcement ratio rl of 2.47%. Deformed 22 mm diameter ties with 135 degree bent hooks lap-spliced with forty times the bar diameter were provided. Outer ties were spaced at 150 mm and inner ties were spaced at 300 mm throughout the column height. Cross-ties were provided at 150 mm spacing within a height of 2.7 m from the column base to increase confinement of the rectangular ties. Tie reinforcement ratio rs within a height of 2.7 m from the column base is 1.72%.

On the other hand, HPF-S column was designed based on 6/35 geometrical scale of C1-6 column. 6/35 refers to the 35 mm nominal diameter of longitudinal bars in C1-6 and 6 mm in HPF-S, respectively. Based on this scale, the column is 1.37 m high from the top of the footing to the loading point (total height from the top of footing to the column top is 1.63 m). The column’s square cross-section dimension was rounded off to 0.3 m to become multiple of 10. PFRC was applied within 0.45 m from the column base. Like C1-6 column, such value was three times the estimated plastic hinge length of 0.15 m. The other portions including the entire footing were built using ordinary concrete. As will be described later, this resulted in more severe column failure at the column base. PFRC of the same mix design, fiber volume and fiber properties as C1-6 column was used in the scaled model. The nominal strength of PFRC was 40 MPa while that of concrete was 30 MPa.

Eighty deformed longitudinal bars with diameter of 6 mm were also set in two layers in the scaled model corresponding to a reinforcement ratio rl of 2.62%. Deformed bars with a diameter of 4 mm were set at every 26 mm for outer ties and at every 52 mm for inner ties. Four cross ties were provided at every 26 mm in the column within 450 mm from the base while two cross ties were provided every 26 mm in the column higher than 450 mm from the base. Volumetric tie reinforcement ratio rs in the model column was 1.90%. Note that discrepancies between the columns were avoided as much as possible. However, constraint in the availability of construction materials (e.g. diameter of steel bars) and rounding-off of dimensions cannot be avoided.

2.2. Loading Condition

Fig. 3 shows the shake-table experiment set-up for C1-6 column while Fig. 4 shows the quasi-dynamic loading (seismic response loading) set-up for HPF-S column. C1-6 column was subjected to shake-table excitations using the E-Defense shake table, the world’s largest shake table constructed by the National Research Institute for Earth Science and Disaster Prevention (NIED), Japan. This shake table is a three-dimensional, full-scale earthquake simulator measuring 20 m long by 15 m wide.

The column was excited using the near-field ground motion recorded at the JR Takatori Station during the 1995 Kobe earthquake. Taking into account the effect of soil-structure interaction, a ground motion with 80% of the original intensity of the JR Takatori record was imposed as a command to the table in the experiment. This ground motion is called the 100% E-Takatori ground motion. The EW, NS, and UD components of the 100% E-Takatori ground motion were applied in the longitudinal, transverse, and vertical directions, respectively, of the bridge model.

As summarized in Table 1, shake table excitations were conducted six times. For the first two excitations, the column was subjected to the 100% E-Takatori ground motion under a deck mass of 307 t. Then, two additional mass blocks were applied corresponding to 21% mass increase (307 tf to 372 tf). The column was once excited under the same intensity. The last three repeating excitations corresponded to 25% increase in the amplitude of the E-Takatori ground motion. Note that 125% E-Takatori ground motion was the strongest ground motion that can be generated by the E-Defense shake table due to stroke limitations.

HPF-S column was loaded similar to the sequence of C1-6 column's shake table excitations. Moreover, the scaled column was loaded until 9th excitation. Note that comparison of response between the full-scale and scaled model will be made only until the 6th excitation.

Mass-based similitude law considering strength of materials was employed but neglecting the effect of gravity loads. In this law, the scale factor for mass must be (6/35)3 [14] to accurately simulate the mass distribution of C1-6 column. Since there was a constraint in loading rate (i.e. the scale factor for the duration is 10 instead of 6/35), the dimensional analysis has been modified so that the necessary scale factor for force (i.e. (6/35)2) was still obtained.

Fig. 3 C1-6 column experiment set-up using E-Defense shake table

Fig. 4 HPF-S column experiment set-up

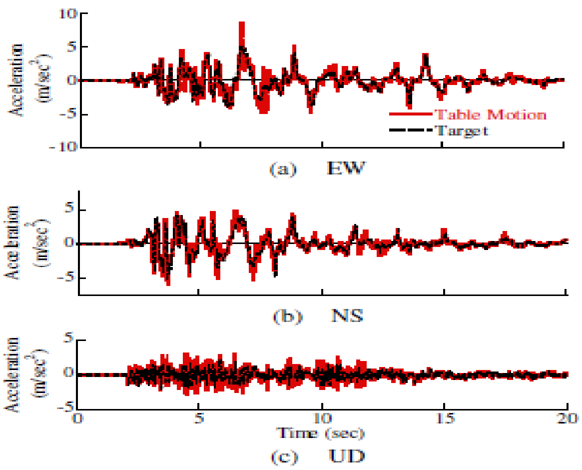


Fig. 5 E-Takatori ground motion

ISBN : 978-602-72437-2-9

Table 1. Sequence of shake table excitations for C1-6 column.

Loading Sequence	Mass of Deck (tf)	Intensity of Excitation
1	307 (3012 kN)	100%
2		100%
3	372 (3649 kN)	100%
4		125%
5		125%
6		125%

In the HPF-S column, the lateral response displacements measured at the top of C1-6 column were imposed at a scale of 6/35 to HPF-S column at the loading point using displacement control. Moreover, the vertical force that acted on the base of the prototype was also imposed at a scale of (6/35)2 to the top of HPF-S column using force control. Such command was modified to account the P- effect, effect of rotation and sliding of the footing, and geometric interactions within the three components of loading displacements [13]. Such a loading is referred here as "seismic response loading."

3. Effect of PFRC on Column Seismic Performance

3.1. Damage Progress of C1-6 Column

Fig. 6 shows the damage progress of C1-6 column within 1.2m from the column base at the NE and SW cornerduring the 6th excitation, the last excitation, when the response displacement was at the peak. At the peak response displacement, the NE corner was subjected to tension whereas the SW corner was subjected to compression.

During the first excitation, at a peak drift of 1%, only micro cracks were observed around the column. During the second excitation, a few flexural cracks were observed where a maximum crack opening of about 8 mm at a height of 0.60 m from the column base occurred at the NE corner. On the other hand, vertical hairline cracks started to widen at the NE and SW corners. The damage progressed during the 6th excitation. At a peak drift of 6%, PFRC cover did not crush however flexural cracks propagated all around the column. From the video records, maximum flexural crack opening of about 20 mm was determined at the NE corner and 10 mm at a height of 1.2 m from the column base at the SW corner during the excitation. The vertical crack at the SW corner progressed and opened with a maximum width of approximately 14 mm. In general, local bar buckling was limited. Note that even after six times of excitation, the damage sustained by C1-6 column was much less than the damage of regular reinforced concrete columns.

Fig. 6 Damage progress of C1-6 at the NE and SW corner during the 6th shake-table excitation

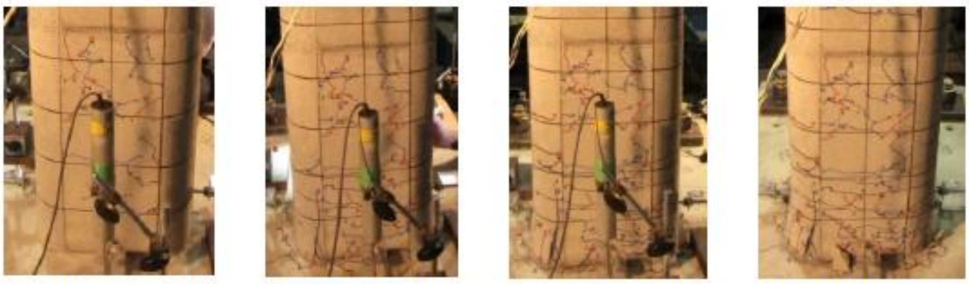
3.2. Damage Progress of HPF-S Column

Fig. 7 shows the damage progress of HPF-S column after the 6th excitation. During the 1st, 2nd, and 3rd excitation at the peak drift of 1.1%, 1.2% and 1.9%, respectively, only flexural cracks as wide as 0.06 mm to 0.15 mm occurred. During the 4th excitation, the peak lateral drift of the column was 3.7 % drift, and a major vertical crack occurred at the SW corner which extended during the 5th excitation (the peak lateral drift was 5.1 % drift). Note that at the 4th excitation, concrete at the footing surface crushed as deep as 30 mm and local buckling of longitudinal bars later developed at this location.

During the 6th and 7th excitations (the peak lateral drift was 6.0 % and 7.5 % respectively), the vertical cracks further extended however PFRC did not suffer extensive damage without deterioration of the flexural restoring force. During the 8th excitation, the peak column displacement reached 9.0% drift, and at least four longitudinal bars ruptured. After the 9th excitation, the flexural restoring force deteriorated.

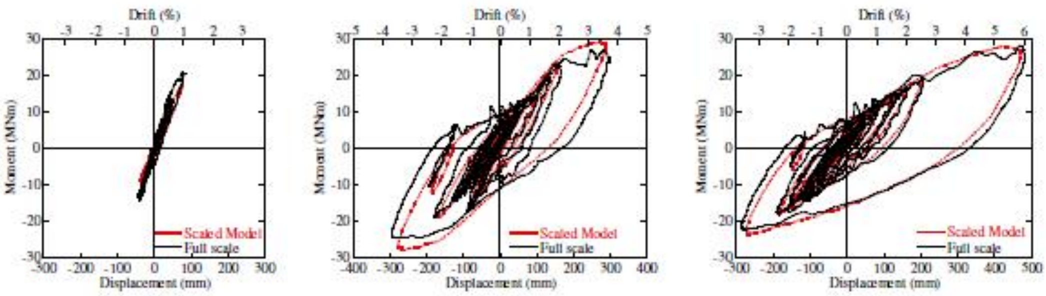
3.3. Moment and Ductility Capacity

Fig. 8 shows the alignment of the moment-lateral displacement/drift hystereses between C1-6 column and HPF-S column (scaled up by (35/6)3) in the principal response direction, i.e. the direction where the response displacement is at maximum. The HPF-S column had provided about 8% higher moment capacity than C1-6 column during the 3rd excitation. This observation is similar to the study of Oya et. al [17] wherein they investigated the behavior of a fullscale and scaled model reinforced concrete column subjected to shake-table experiments and seismic response loading experiments, respectively. The difference in moment capacity is attributed to the slower progress of failure of the model column compared to the full-scale column. As such, C1-6 column experienced larger drift than HPF-S column during the entire series of excitations.



(a) After 3rd excitation (b) After 5th excitaion (c) After 8th excitation (d) After 9th excitation

Fig. 7 Damage progress of HPF-S column under seismic response loading



(a) 1st excitation (b) 3rd excitation (c) 6th excitation

Fig. 8 Damage progress of HPF-S column under seismic response loading

ISBN : 978-602-72437-2-9

4. Conclusions

Based on the results, it was observed that there is a strong similarity between the full-scale and model column's damage progress and moment vs. displacement/drift capacity. However, it was observed that the progress of damage was much slower in HPF-S model column compared to the C1-6 full-scale column. This was evident in the 8% overestimation in moment capacity of the model column. This shows that the scale factor of time in similitude procedure has great effect in prediction of column's response.

5. References

[1] M.J.N. Priestley, G. M. Calvi, M. J. Kowalsky, Displacement-based Seismic Design of Structures. IUSS Press, Pavia, 2007.

[2] M.J.N. Priestley, F. Seible, G. M. Calvi, Seismic Design and Retrofit of Bridges. John Wiley & Sons, New York, 1996.

[3] V. C. Li, Engineered cementitious composites (ECC) – tailored composites through micromechanical modeling. In N. Banthia and A. Mufti (Eds.), Fiber Reinforced Concrete: Present and the Future, Canadian Society of Civil Engineers, Montreal, 1998, 64–97.

[4] T. Matsumoto, H. Mihashi, DFRCC terminology and application concepts, J. Adv. Conc. Tech., JCI 1(3) (2003) 335–340.

[5] Parra-Montesinos, G., High-performance fiber-reinforced cement composites: an alternative for seismic design of structures, ACI Struct. J. 102(5), (2005) 668–675.

[6] K. Kawashima, R. G. Zafra, T. Sasaki, K. Kajiwara, M. Nakayama, Effect of polypropylene fiber reinforced cement composite and steel fiber reinforced concrete for enhancing the seismic performance of bridge columns, J. Earthq. Eng. 15 (2011) 1194–1211.

[7] C. Cruz Noguez, M. Saiidi, Shake table studies of a 4-span bridge model with advanced materials, J. Struct. Eng., ASCE 138(2) (2012) 183–192.

[8] H. Fukuyama, H. Suwada, Experimental response of HPFRCC dampers for structural control, J. Adv. Concr. Tech., JCI 1(3) (2003) 317–326.

[9] K. Kesner, S. Billington, Investigation of infill panels made from engineered cementitious composites for seismic strengthening and retrofit, J. Struct. Eng., ASCE 131(11) (2005) 1712–1720.

[10] K. Kosa, K. Wakita, K., H. Goda, A. Ogawa, Seismic strengthening of piers with partial use of high ductility cement. In C. U. Grosse (Ed.), Advances in Construction Materials, Springer, Berlin Heidelberg, 2007, 269–277.

[11] K. Kawashima, R. G. Zafra, T. Sasaki, K. Kajiwara, M. Nakayama, S. Unjoh, J. Sakai, K. Kosa, Y. Takahashi, M. Yabe. Seismic performance of a full-size polypropylene fiber-reinforced cement composite bridge column based on E-Defense shake table experiments, J. Earthq. Eng. 16(4) (2012) 463-495.

[12] T. Sasaki, R. Zafra, K. Kawashima, Effect of polypropylene fiber reinforced concrete for enhancing the seismic performance of bridge columns, Proc. 15th World Conference on Earthquake Engineering, Lisbon, Portugal, 2012.

[13] Japan Road Association Specifications for Highway Bridges, Maruzen, Tokyo, 2002.

[14] H. G. Harris, G. Sabnis, Structural modeling and experimental techniques (2nd Ed.). CRC Press, Florida, 1999.

[15] T. Oya, K. Ohta, H. Matsuzaki, K. Kawashima, Size effect on the progress of flexural failure of RC bridge columns. J. Japan Soc. Civ. Eng., 69(4) (2013) 829-838. doi: http://dx.doi.org/10.2208/jscejseee.69.I_829.

This page is intentionally left blank